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Carl Bernard Martin

MODEL STUDIES OF THE BEARING CAPACITY OF PILE GROUPS IN A SATURATED CLAY

A THESIS

Presented to

the Faculty of the Graduate Division

by

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MODEL STUDIES OF THE

BEARING CAPACITY OF PILE GROUPS

IN A SATURATED CLAY

Approved:

George F. Sowers

Austin B. Caseman

James E. Sisson

Date Approved by Chairman May 6, 1957

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MODEL STUDIES OF THE
BEARING CAPACITY OF PILE GROUPS
IN A SATURATED CLAY

Carl Bernard Martin
53 Pages
Directed By Professor George F. Sowers

These studies were undertaken to obtain an insight into the action of pile groups in transferring loads into a saturated clay soil and to determine the effects of several factors on the bearing capacity of these groups.

These tests were performed on each of four groups of piles at four different spacings and through a range of three lengths. The bearing capacities of the pile groups were compared to the bearing capacity of a single isolated pile (tested at the same time and at the same length as the pile groups) and to the theoretical bearing capacities.

It was found that pile groups transfer loads into the soil by one of three actions: a unit or box type action, an individual action, or a combination of the two actions.

The ratio of the measured bearing capacity per pile to the computed bearing capacity (using the results of vane shear tests) was found to decrease with an increase in length and to increase with an increase in the number of piles. This ratio decreases with an increase in spacing in the unit action zone and increases with an increase in spacing in the individual action zone. The ratio reaches a minimum

in the intermediate zone.

The results show that the theories available at the present time are not capable of accurately predicting the action of a group of piles. In general the theoretical bearing capacities were found to be very unsafe.

It was shown that the length of a pile or group of piles is not as significant as the length-width ratio. Therefore, it is erroneous to assume that the testing of a single pile (the same length as a group of piles) eliminates any effect due to length when using the bearing capacity of the single pile to determine the bearing capacity of a group of piles.

Before a quantitative analysis can be developed for predicting the bearing capacity of pile groups it will be necessary to understand the distribution of loads among the piles and the stress conditions in the soil. Any further research on this subject should be directed toward these ends.

Approved by: George F. Sowers

Date Approved: May 6, 1957

CHAPTER I

INTRODUCTION

When surface soils are not capable of supporting the loads imposed on them, it is sometimes desirable to use pile foundations. Essentially piles are long shafts, forced into the ground, which transfer the loads to stronger underlying soil or distribute the loads throughout a weak soil.

The increased usage of heavy structures, in recent years, has led to an augmented demand for pile foundations. At the same time the increased cost of labor and materials has made economy an important factor in the design of pile foundations. Efforts to obtain economical designs have necessarily led to a number of attempts to understand the action of piles in transferring the loads into the soil.

At the present time there are a number of theories available. Due to the prohibitive cost of full scale tests the majority of the existing formulas are based on empirical relationships supported by very limited data or speculative theory based on a large number of simplifying assumptions.

The bearing capacity of piles may range from twenty or thirty tons to several hundred tons. Because of the cost involved in supplying loads of this magnitude, the majority of full scale tests have been performed on single piles. As a result the action of single piles is understood much better than the action of groups of piles.

The usual method for estimating the bearing capacity of single piles is based on relationships involving the resistance of the pile to driving. On some large construction projects this bearing capacity is verified by performing full scale load tests on one or more of the piles.

It is generally believed that the bearing capacity per pile for a group of closely spaced piles is less than that of a single isolated pile. The ratio of the single pile bearing capacity to the bearing capacity per pile of the group, expressed as a percentage, is called the efficiency factor. There are a number of formulas available for determining this efficiency factor. However, most of them appear to be based on the intuition of the author and are supported only by a limited amount of data. In general, these formulas are ultra-conservative and do not meet the needs of an economical design.

A common method of determining the bearing capacity of a group of piles is to apply an efficiency factor to the bearing capacity of a single pile as determined by full scale field tests. This is then multiplied by the number of piles in the group.

The factors which have been considered to influence the action of a group of piles are: (a) the number of piles in the group, (b) the spacing of the piles, (c) the length of the piles, (d) the arrangement of the piles or the shape of the pile group, (e) the character of the soil in which the piles are placed, and (f) the rigidity of the pile cap. The last factor would probably be considered a structural problem.

The purpose of this research is to study the factors influencing the bearing capacity of pile groups in clay. Spacing, length and the number of piles were selected as the variable factors to be studied.

All tests were performed in an artifically prepared saturated clay (cohesive) soil. A clay soil is one in which the adsorbed layers of water are thick compared to the size of the individual soil grains. This type of soil has the ability to deform plastically without cracking.

CHAPTER II

THEORY

Piles transfer their loads into the soil by two separate actions. The loads are transferred into the soil by end bearing (direct compression of the soil beneath the pile point) and by shear or skin friction along the sides of the piles. While some piles transfer their loads into the soil by only one of the above actions, the majority of piles employ a combination of the two actions.

The portion of the load transferred into a cohesive soil through end bearing of a single footing (1) may be expressed by

$$P_b = (q' + 4c)A \qquad \text{(Eq. 1)}$$

where P_b = the load transferred in bearing
q' = the surcharge
c = the cohesion or the shear strength
A = the bearing area

Under ordinary conditions

$$q' = GL \qquad \text{(Eq. 2)}$$

where G = the unit weight of the soil
L = the length of the pile or the depth to the point

Therefore, by substituting Equation 2 in Equation 1 the end bearing is

$$P_b = (GL + 4c)A \qquad \text{(Eq. 3)}$$

The portion of the load transferred into a cohesive soil by shear or skin friction is (2)

$$P_s = cS \qquad \text{(Eq. 4)}$$

where P_s = the load transferred in shear
S = the surface area

The load transferred into a cohesive soil through a combination of end bearing and shear is

$$P = P_b + P_s \qquad \text{(Eq. 5)}$$

which, by substituting Equation 3 and Equation 4 becomes

$$P = (GL + 4c)A + cS \qquad \text{(Eq. 6)}$$

From Equation 6 the bearing capacity of a group of piles acting individually would be

$$P = N\ [\ (GL + 4c)A + cS\ ] \qquad \text{(Eq. 6a)}$$

where N = the number of piles in the group

It has been observed that when piles are closely spaced they act as a unit rather than as a number of individual piles. This action is similar to the action of a single large pier having the same dimensions as the periphery of the group of piles. The bearing capacity of a group of piles, when acting in this manner, can be found from Equation 6.

$$P = (GL + 4c)A_g + cS_g \qquad \text{(Eq. 7)}$$

where A_g = the end area of the group
S_g = the surface area of the periphery of the group

In Equation 7 the surcharge term is GLA_g. This entire mass of soil has not been removed so that if this term is considered as a surcharge the weight of the material remaining between the piles must be considered as part of the load. Equation 7 would then appear as

$$P = P_L + P_w + GL(A_g - A_p) = GLA_g + 4cA_g + cS_g \qquad \text{(Eq. 8)}$$

where P_L = the load applied to the piles at failure
P_w = the weight of the pile group
A_p = the sum of the point areas of the piles

The weight of the piles would be

$$P_w = G_p L A_p \qquad \text{(Eq. 9)}$$

where G_p = the unit weight of the piles

If it is assumed that the unit weight of the piles is approximately equal to the unit weight of the soil, Equation 8 becomes

$$P = 4cA_g + cS_g \qquad \text{(Eq. 10)}$$

It is reasonable to assume that, at some spacing wide enough, each pile in a group will act independently of the other piles. At very close spacings the piles will act as a unit. There must then be a point of transition from one type of action to the other. It has been suggested that this transition occurs, with long friction piles, when the perimeter of the group is equal to the sum of the perimeters of the piles in the group (3).

Recent tests by G. G. Meyerhof (4) of various types of foundations indicate that the end bearing of foundations in a cohesive soil is independent of the depth at depths exceeding two times the width. This bearing capacity was found to be

$$P_b = 9cA \qquad \text{(Eq. 11)}$$

The same tests found the shear along the sides to be constant with depth. The bearing capacity based on these results is

$$P = 9cA + cS \qquad \text{(Eq. 12)}$$

It is noted that these results are based on a maximum length-width ratio of 14.

A common method for determining the bearing capacity of a group of piles is to apply an empirical efficiency factor to the load supported by a single isolated pile times the number of piles in the group. The single pile bearing capacity is usually determined from full scale field tests. There are a number of formulas available for determining the single pile efficiency of a group of piles.

The Converse-Labarre method (5) for determining the single pile efficiency of a group of piles is

$$\text{Efficiency} = 1 - \phi \, \frac{(n - 1)m + (m - 1)n}{90mn} \qquad \text{(Eq. 13)}$$

where
m = the number of rows
n = the number of piles per row
$\phi = \tan^{-1} d/s$ (in degrees)
s = the center to center spacing
d = the pile diameter

The Seiler-Keeney method (6) for determining the single pile efficiency of a group of piles is

$$\text{Efficiency} = \left[1 - \frac{11s}{7(s^2 - 1)} \times \frac{(m + n - 2)}{(m + n - 1)} \right] + \frac{0.3}{m + n} \qquad \text{(Eq. 14)}$$

The Feld method (7) has been suggested for obtaining a rough estimate of the single pile efficiency of a group of piles. It consists of reducing the efficiency of each pile one-sixteenth due to the effect of each adjacent pile, both in line and diagonally adjacent. For example a square, five-pile group will have four piles acting at 13/16ths of the capacity and one pile acting at 12/16ths of the capacity. This gives a single pile efficiency for the group of 80 percent.

It is noticed that none of the above formulas consider the effect of length. The methods assume that when testing the single pile at the same length as the group of piles any effect due the length has been eliminated.

At the present time the majority of the building codes consider the design of piles only by specifying a minimum spacing. The minimum spacing is usually expressed in pile diameters but some codes specify the minimum spacing in feet. Typical minimum values are two to three diameters or about two and a half feet. These codes do not specify any reduction due to a consideration of group action.

CHAPTER III

APPARATUS

Model piles were made from one-half inch diameter aluminium rods cut into 24-inch lengths (Fig. 10). The rods were threaded at one end to provide for easy insertion into three-quarter inch thick, threaded plywood pile caps. At the small spacings it became difficult to thread the pile caps accurately; therefore, cast pile caps were used. The pile caps were cast of Brimsto, a sulphur-base, thiokol-plasticized compound used for capping concrete test cylinders.

Commercial bentonite was selected to be used as a saturated clay. Bentonite is a highly colloidal, sensitive plastic clay which, when mixed with water, forms a thixotropic gel and swells to several times its original size. The gel of bentonite and water is highly sensitive but posesses the ability to re-gel after being remolded. This property is very desirable as it enables the same mass of clay to be used throughout a lengthy testing program.

Several small samples of bentonite were prepared at different water contents and a water content of 300 percent was selected to be used throughout the tests.

The bentonite and the proper amount of water were placed in large mixing pan. The mixture was allowed to stay in these large pans for about one week. During this time the bentonite was vigorously mixed once each day. At the end of a week the mixture was squeezed and kneaded, by hand, through a number four sieve and placed in three

55-gallon drums(approximately two feet in diameter and three feet high). The drums had been previously cleaned and painted with aluminum paint to prevent rusting. They were then tightly covered with sheets of polyethylene to prevent moisture loss. The drums remained covered at all times except during the testing.

A miniature vane was constructed to determine the shear strength of the bentonite throughout the tests (Fig. 4 and 11). The blades of the vane were sharpened and polished to minimize the disturbance that occurs during the insertion of the vane into the soil. Bearings were appropriately placed on the shaft of the vane to minimize any mechanical friction that might exist. During the later stages of testing, an extension was made for the vane to allow shear tests to be made at various depths.

Lead weights, of various sizes, were used to load the piles and a standard type micrometer dial gage was employed in making settlement observations (Fig. 12).

CHAPTER IV

PROCEDURE

Preliminary tests were performed on single piles to determine the time required for the bentonite to re-gel after having been disturbed or remolded. It was found that about three days were required for the bentonite to re-gel after being disturbed by loading a pile to failure. The time required for the bentonite to re-gel after being completely remolded was found to be about five days. On the basis of these tests it was decided to proceed with the testing on a weekly cycle.

Pile groups were tested containing two, four, nine, and sixteen piles. With the exception of the two-pile group all pile groups were square. The two-and the four-pile groups were considered small enough to be placed in the same drum. The nine-and the sixteen pile groups were each placed in separate containers. The pile groups were tested at lengths of 12, 24 and 36 pile diameters. The spacings used were 1.50, 1.75, 2.00, and 3.00 diameters. One exception to these spacings was that the sixteen-pile group was tested at a spacing of 2.50 diameters instead of 1.75 diameters (Table 3a).

The general test procedure consisted of setting a spacing and pressing the pile into the clay to a length of 12 diameters. The piles were allowed to stand a week before testing. After the piles were tested they were pressed into the clay to a depth of 24 diameters.

At the end of another week the piles were tested again. They were then pressed into the clay to a depth of 36 diameters and after a week tested the third time. The piles were then removed from the drums. At this time the bentonite was remixed to remove the holes left by the piles. A different spacing was selected and the weekly testing cycle repeated.

The piles were loaded by simply stacking lead weights on the pile caps. Special care was taken to avoid any impact. The load increments were decreased as the piles approached the failure load to obtain a more exact point of failure. Settlement measurements were made by placing a micrometer dial gage directly on the pile caps and observing the settlement under each load increment.

Each week, at the time the tests were performed on the pile groups, a single isolated pile was tested in each drum. The single piles were tested at the same lengths as the pile groups. Also a vane shear test was performed in each container at the same time the other tests were performed.

The tests were performed on each of four groups of piles at four different spacings and through a range of three lengths. A typical example of the test data is shown in Table 4.

CHAPTER V

RESULTS

The shear strength of the bentonite, as determined by the vane shear test, is shown with respect to time (Fig. 13). It is noticed that the shear strength did not vary an appreciable amount throughout the entire testing program. The curves are broken every three weeks because the bentonite was remixed at these times. It is noticed that any pronounced changes occurred at these times can be due to to the remolding of the bentonite. Other minor variations in the curves are probably due to non-uniformity within the mass of clay.

In the case of the single piles the failure load was calculated from Equation 6 and Equation 12 by using the shear strength determined from the vane shear tests. The ratios of the measured failure load to the calculated failure loads, expressed as percentages, are averaged and compared to the length (Fig. 1). Trends shown could indicate that the end bearing portion of the load is not constant with depth or that the shear along the sides is not constant with depth. The curves of Fig. 15 show the settlement at failure of the single piles. There is a general tendency for the settlement at failure to increase with an increase in length. The curves of Fig. 1 and Fig. 15 when considered together indicate that the portion of the load transferred in shear is not constant with depth. This does not indicate that the shear strength is not constant but is interpreted as indicating a progressive type shear failure due to unequal strains.

The two curves on Fig. 1 are practically identical for the two methods of analysis. This is due to the relative unimportance of end bearing, in saturated clay, of piles with a high length-width ratio. An extrapolation of the curves to a zero length shows that the measured bearing capacity would be between 1.1 and 1.2 times the bearing capacity of surface footings, indicated from the results of the vane shear test. These values compare very favorably with the results of previous studies, by the author, of various spread footings (8). The curves indicate that the ratio of the measured failure load to the calculated, based on data from the the vane shear tests, decreases with a increasing length and approaches a constant value of 60 - 65 percent at lengths between 45 and 50 diameters.

The theoretical failure load of each of the pile groups was calculated by assuming an individual action of the piles (Equation 6a) and by assuming a unit or box type action (Equation 7). The ratios of the measured failure load per pile to the calculated failure load per pile are expressed as percentages. These percentages will hereafter be referred to as the theoretical individual action efficiency and the theoretical unit action efficiency. In addition to the above ratios the ratio of the measured failure load per pile to the failure load of a single isolated pile (the same length and in the same container) is used as a basis of comparison. This ratio will be expressed as a percentage and referred to as the single pile efficiency.

While the results are all expressed as efficiencies, it should be pointed out that the use of the term "efficiency" does not indicate that the bearing capacities are not as they should be. This indicates that the formulas and theories in use have not been sufficiently developed to include the effects of all the factors that influence the bearing capacity of pile groups. The single pile efficiency is probably the only legitimate efficiency term used.

The theoretical unit action and individual action efficiencies are compared to the spacing for each of the pile groups at each length (Fig. 2). It is observed that for all except the two-pile group the unit action curve crosses the individual action curve. This point of intersection should indicate the spacing at which the dominating action changes from a unit action to an individual action. It follows then that the uppermust curves are the best approximations available at the present for determining the bearing capacity of groups of piles. The points on the curves that could be drawn following the uppermost curves will be referred to as theoretical efficiencies. It is noticed then , that the critical spacing occurs at the point of minimum theoretical efficiency. The two-pile group and the sixteen-pile group did not show such marked tendencies, but it is noted that the two-pile group was not tested at small enough spacings to indicate a critical spacing, and the sixteen-pile group indicated critical spacings so near the end of the curve that the individual action portion is not included. The deviation from the general trend, by the sixteen-pile group, in the region of the critical spacing, could be due to the fact that tests were not performed at sufficiently large spacings to include this trend. This deviation could also be due to factors not included in this study such as the rigidity of the pile cap or stress interferences with the sides of the container used in the tests.

It can be seen that the critical spacing indicated by the efficiency-spacing curves increases with an increase in length (Fig. 3). It is noted that the critical spacing determined from the efficiency-spacing curves is greater than the theoretical critical spacing for small pile groups and less than the theoretical spacing for the large pile groups. It is interesting to note that the curves for the critical spacing, when extended, all intersect at a spacing of one diameter, which is the smallest spacing at which piles could be placed.

At the smaller spacings the unit action failure was visible (Fig. 4). The mass of soil included within the perimeter of the pile group sheared along the sides and moved down with the piles. The spacing at which this unit action was visible is compared to the other critical spacing curves (Fig. 3), and it is seen that the observed critical spacings were lower than those indicated by any other method. This indicates that at the observed critical spacing the action of the pile group was entirely the unit action.

Since the tests have verified the existence of a unit action and it is reasonable to assume, that at some spacing large enough, there is a purely individual action, the variations of the critical spacings indicated by the different methods suggests the presence of a zone in which there is a combination of the two types of actions. The theoretical efficiency curves (Fig. 2) show the lowest values in the region of the critical spacing; therefore, in this transition zone the two actions must interfere with each other, resulting in a lowered bearing capacity. It is unfortunate that this interference and the resulting lowered bearing capacity occur in the range of spacings that are commonly used in engineering practice.

The single pile efficiency is compared to the spacing (Fig. 5). It is noted that these efficiency curves show a very noticeable dip at the smaller spacings followed by a sharp rise and a gradual decrease with increasing spacings. This dip and the accompanying rise is more pronounced at the 24 diameter length. The curves for the sixteen-pile group did not show this dip, but since tests were not performed at the 1.75 diameter spacing, it is possible the dip would have occurred. The dip can be best explained by considering the effect of the length-width ratio (Fig. 1). It has been shown that at the samll spacings, where the dip occurs, the pile groups are acting as a unit. With this type of action the group must be considered as a single large pile for determining the length-width ratio. Considering the group as a single pile would result in a low length-width ratio compared to the length-width ratio of a single pile. The effect of the length-width ratio can be seen in Fig. 1. The difference in the length-width ratios will, therefore, be reflected in the single pile efficiency curves and would indicate a low efficiency. As the spacing increases, the action of the piles enters the zone where there is a combination type action. This means that the effect of the individual action is becoming gradually more dominant. The introduction of the individual action effect has the same effect on the pile group as increasing the length-width ratio. An increase in the length-width ratio of the group would result in a decrease in the difference in the length-width ratios between the group and the single pile. From Fig. 1 it is seen that a decrease in the difference of the length-width ratios would be reflected as an increase in the single pile efficiency curves. The gradual decrease in the single pile efficiency at the larger spacings is probably due to the interference and overlapping of the stress patterns around each individual pile.

In general the single pile efficiency curves, when considered with respect to the curves of Fig. 1, indicate that the length-width ratio should be considered in an analysis rather than the length alone. This also indicates an error in the assumption that a full scale load test on a single pile (the same length as a group of piles) will eliminate any effect due to length when using the bearing capacity of the single pile in the design of a group of piles.

The theoretical unit action and individual action efficiencies are compared to the pile length for each pile group and each spacing (Fig. 6). It is shown that these efficiencies generally decrease with an increase in length. This appears to be due to a progressive type of shear failure along the sides. It is observed that the curves for any one pile group will get progressively closer with an increase in spacing and finally cross over, after which they move farther apart. This crossover point is the critical spacing point observed on the efficiency-spacing curves. It is noticed that the efficiency at the

1.75 diameter spacing increases with an increasing length. This is due to the same action that caused the dip in the efficiency-spacing curves.

The single pile efficiency curves when compared to the length show a slight increase in efficiency with an increase in length (Fig. 7). This indicates an error in the assumption that testing a single isolated pile at the same length as a group of piles will eliminate the length effect. These curves also indicate that the efficiency is dependent on the ratio of the length to width instead of the length alone.

The theoretical unit action and individual action efficiencies are compared to the number of piles for each length and each spacing (Fig. 8). It can be seen that there is an increase in the efficiency with an increase in the number of piles. The point of intersection of these curves should indicate the number of piles at a particular spacing and length that are required to cause a change in the type of action the piles employ to transfer their loads. The curves become increasingly flatter as the spacing increases until at the larger spacings the effect of the number of piles is very slight.

The single pile efficiency curves (Fig. 9) when compared to the number of piles indicate a decrease in efficiency with an increase in the number of piles.

An attempt was made to analyze the pile groups on the basis of the Meyerhof method (Equation 12). It is obvious by inspection that this method will give lower efficiencies for an individual action analysis than the conventional method. The Meyerhof analysis, however, gave very good results at spacings at or below the observed critical spacing (Table 2a). To obtain these results it was necessary to compute the surface area and the point area based on the assumption that the perimeter of the groups lay on the lines connecting the centers of the outside piles. This assumption is incorrect since it was observed that the surface of shear was on the outer perimeter of the pile group when a unit action failure occured.

The single pile efficiency of the pile groups was computed from the formulas of Equation 13 and Equation 14 and compared to the measured single pile efficiencies. The Sieler-Keeney formula yielded negative efficiencies in the case of the small spacings and very low efficiencies at the larger spacings. This illustrates the limitations of some of the existing formulas. The Converse-Labarre formula yielded efficiencies that were approximately equal to the measured efficiencies for the small pile groups at the small spacings but were progressively more conservative with the larger groups. This calculated efficiency became more conservative with an increase in length but became unsafe at some of the larger spacings (Table 1a). The wide range of differences between the measured efficiency and the calculated efficiency again illustrates the limitations of the existing empirical formulas.

It was observed during testing that some of the smaller pile groups displayed very erratic settlement characteristics near the failure load, and often failure was quite sudden. This indicates that possibly one or more of the piles failed before the entire group had reached the failure load. The local failure of one or more piles transferred the loads to the other piles in the group, overloading the remaining piles and causing the entire group to fail.

The settlements of the pile groups at failure was compared to the spacing, the number of piles and the length (Fig. 16). The results shown by the settlement curves are very erratic but the general trends are visible. The settlement at failure tends to decrease with an increase in the spacing. The comparison of settlement at failure with the length indicates a slight increase in settlement with an increase in length. The settlements when compared with the number of piles, show a very pronounced tendency to increase with an increase in the number of piles. In observing the load settlement curves it was found that the bentonite was practically elastic, almost to failure.

At one point during the testing an attempt was made to determine the resistance of the pile groups to pulling. These results were unsatisfactory since several times the compressive failure load was required to pull the piles. This was probably due to the vaccuum created at the pile points.

At the end of the testing it was considered desirable to discover if there was a variation in shear strength with depth in the bentonite. An extension was made for the vane shear device and shear tests were performed in each container of bentonite at three different depths (Table 2b). In one case the shear strength did not vary with depth, but in two other cases there was a slight increase with depth. This increase could possibly be due to shear acting along the vane shaft and not due to an actual increase in shear strength.

CHAPTER VI

CONCLUSIONS

The following conclusions have been drawn as a result of these studies. These conclusions should be interpreted as pertaining to a saturated sensitive clay soil only.

1. The end bearing of a pile or groups of piles is constant with depth (for long piles) and a progressive type shear failure takes place along the sides of the pile or pile group.

2. Pile groups transfer their loads into the soil by one of three types of actions: a unit or box type action, an individual pile action and a combination of the two actions.

3. In general the theoretical efficiency of any given pile group will decrease with an increase in spacing from the group action into the range of a combination action where it reaches a minimum. From this point the theoretical efficiency increases with an increase in spacing as the individual action becomes more dominant.

4. The theoretical efficiency of a given group decreases with an increase in the length.

5. The theoretical efficiency of a given pile group increases with an increase in the number of piles.

6. The length of a pile or group of piles expressed as a ratio of the length to the width has more significance in the analysis than the length expressed alone in some units.

7. The assumption that the testing of a full scale pile (the same length as a group of piles) eliminates any effect due to length when using the bearing capacity of the single pile in the design of a pile group is erroneous.

8. The settlement of a given pile group tends to decrease with an increase in the spacing, increase slightly with an increase in length, and to increase with an increase in the number of piles.

9. The available efficiency formulas, while extremely conservative in most instances, will at times yield results that are unsafe.

10. The theoretical critical spacing gives values in most instances that are too high. The critical spacing increases with an increase in the length.

11. There is, at times, a local failure of one or more piles in a small pile group which by transferring the loads to the remaining piles may cause the group to fail.

12. The theoretical bearing capacities of pile groups were found, in most instances, to be unsafe. The magnitude of the error depends on the other factors influencing the action of pile groups.

CHAPTER VII

RECOMMENDATIONS

1. The most pressing need in determining a method for designing pile foundations is an accurate theory for determining the stress distribution around a pile or a group of piles. It is recommended that this be studied.
2. Each of the factors affecting the bearing capacity of pile groups needs to be isolated and studied quantitatively. The main factors that should be studied are the spacing, the number of piles, the length, the shape and arrangement of the piles, and the distribution of the load to each individual pile in a group.
3. An effort should be made to obtain a less sensitive soil for use in any future tests. A combination of bentonite and gelatin may prove to be satisfactory.
4. To obtain a more accurate determination of the shear strength it is suggested that the vane shear test be performed at several locations and depths to average any effect due to nonuniformity.
5. The cast pile caps used with the smaller spacings proved to be more satisfactory than the threaded wooden pile caps since they provided a more uniform spacing throughout the length of the piles. Plaster of Paris is suggested for use instead of Brimsto since the Brimsto tended to shrink and leave a very rough surface.
6. Due to a slight reaction between the aluminium piles and the bentonite, it is suggested that if aluminium piles are used in any future work they be given a protective coating.

A P P E N D I X

TABLE 1

Summary of Test Results and Computed Data

List of Symbols

P_s = Measured load on a single pile

P_g = Measured load on a single pile

P_I = Computed load per pile (individual action)

P_m = Computed load on a single pile (Meyerhof analysis)

$\bar{P}_g$ = Measured load per pile on pile groups

$\bar{P}_{cg}$ = Computed load per pile (group action)

Code

No. Piles)
Length) expressed as (2-12-1.50) means
Spacing)

Two pile group, 12 diameter length, and

1.50 diameter spacing.

Table 1 Continued

Summary of Test Results and Computed Data

Week	Drum No.	No. Piles Length Spacing	P_s (#)	P_g (#)	P_I (#)	P_m (#)	$\bar{P}_g$ (#)	$\bar{P}_{cg}$ (#)
1	1	2-12-2.00	6.90	11.85	9.22	9.57	5.93	12.84
"	"	4-	6.90	22.67	9.22	9.57	5.67	9.63
"	2	9-	5.90	50.43	9.22	9.57	5.60	8.08
"	3	16-	6.23	92.83	9.22	9.57	5.80	7.12
2	1	2-24-2.00	8.91	17.69	17.79	17.64	8.85	24.40
"	"	4-	8.91	35.01	17.79	17.64	8.75	17.33
"	2	9	9.24	76.43	17.79	17.64	8.71	13.79
"	3	16-	11.91	149.83	16.61	16.40	9.36	10.80
3	1	2-36-2.00	11.23	23.03	25.47	24.80	11.52	36.69
"	"	4-	11.23	41.01	25.47	24.80	10.25	24.16
"	2	9-	12.23	99.43	27.25	26.63	11.05	20.20
"	3	16-	16.51	207.83	22.83	22.07	12.99	13.83
4	1	2-12-3.00	7.57	14.03	8.92	9.34	7.02	14.04
"	"	4-	7.57	25.01	8.92	9.34	6.25	13.21
"	2	9-	8.90	59.43	8.61	8.90	6.60	11.85
"	3	16-	9.57	127.83	9.86	10.27	7.99	11.60
5	1	2-24-3.00	11.57	22.03	16.61	16.40	11.02	25.47
"	"	4-	11.57	40.01	16.61	16.40	10.00	22.29
"	2	9-	13.57	89.43	17.22	17.04	9.94	20.00
"	3	16-	15.57	152.83	19.01	18.92	9.55	19.51

Table 1 Continued

Summary of Test Results and Computed Data

Week	Drum No.	No. Piles Length Spacing	P_s (#)	P_g (3)	P_I (#)	P_m (#)	$\bar{P}_g$ (#)	$\bar{P}_{cg}$ (#)
6	1	2-36-3.00	17.57	25.03	23.70	22.97	12.52	35.95
"	"	4-	13.57	48.01	23.70	22.97	12.00	30.60
"	2	9-	15.57	114.43	24.59	23.89	12.71	26.71
"	3	16-	18.57	207.83	27.27	26.65	12.99	25.53
7	1	2-12-1.50	8.75	14.50	7.99	8.22	7.25	8.65
"	"	4-	8.75	24.00	7.99	8.22	6.60	6.66
"	2	9-	8.75	52.10	8.91	9.23	5.79	5.87
"	3	16-	8.75	81.70	9.84	10.25	5.13	5.73
8	1	2-24-1.50	11.75	22.50	15.42	15.15	11.25	16.36
"	"	4-	11.75	39.00	15.42	15.15	9.75	12.17
"	2	9-	10.75	76.50	17.79	17.64	8.50	10.15
"	3	16-	11.75	126.70	18.87	19.55	7.92	9.62
9	1	2-36-1.50	15.75	29.50	24.57	23.87	14.75	26.04
"	"	4-	15.75	51.00	24.57	23.87	12.75	19.13
"	2	9-	15.75	96.50	23.70	22.97	10.72	13.60
"	3	16-	17.75	171.70	23.70	22.97	10.73	10.91
10	1	2-12-1.75	7.75	11.15	9.54	9.91	5.58	11.21
"	"	4-	7.75	23.33	9.54	9.91	5.58	8.99
"	2	9-	7.75	50.07	7.36	7.53	5.56	5.55

Table 1a

Summary of Test Results and Computed Data

No. Piles Length Spacing	P_s/P_I (%)	P_s/P_m (%)	$\bar{P}_g/P_s$ (%)	$\bar{P}_g/\bar{P}_{cg}$ (%)	$\bar{P}_g/P_I$ (%)	Converse Labarre Eff. (%)	Single Pile Settlement (in.)	Group Pile Settlement (in.)
2-12-1.50	110	106	83	84	91	81	.015	.016
4-			69	90	75	63		.044
9-	98	95	66	99	65	50	.014	.062
16-	89	85	58	93	52	44	.012	.035
2-24-1.50	76	78	96	69	73	81	.017	.030
4-			83	80	63	63		.031
9-	60	61	79	80	48	50	.011	.091
16-	62	60	67	82	42	44	.013	.054
2-36-1.50	64	65	94	57	60	81	.014	.054
4-			81	67	52	63		.053
9-	66	69	68	79	45	50	.019	.078
16-	75	77	60	98	45	44	.016	.046
2-12-1.75	81	78	72	50	58	83	.015	.015
4-			72	62	58	68		.035
9-	105	103	72	100	76	56	.012	.065
2-24-1.75	77	78	83	54	64	83	.017	.029
4-			79	69	61	68		.042
9-	90	92	79	104	70	56	.008	.033

Table 1a Continued

Summary of Test Results and Computed Data

No. Piles Length Spacing	P_s/P_I (%)	P_s/P_m (%)	$\bar{P}_g/P_s$ (%)	$\bar{P}_g/\bar{P}_{cg}$ (%)	$\bar{P}_g/P_I$ (%)	Converse Labarre Eff. (%)	Single Pile Settlement (in.)	Single Pile Settlement (in.)
2-36-1.75	72	75	90	57	64	83	.015	.032
4-			89	75	64	68		.054
9-	72	75	81	90	58	56	.021	.070
2-12-2.00	75	72	86	46	64	85	.019	.020
4-			82	59	61	71		.040
9-	64	62	95	69	61	61	.009	.069
16-	68	65	93	81	63	56	.008	.130
2-24-2.00	50	51	99	36	50	85	.008	.037
4-			98	50	49	71		.036
9-	52	52	94	63	49	61	.011	.064
16-	72	73	79	87	56	56	.018	.125
2-36-2.00	44	45	102	33	45	85	.106	.019
4-			91	42	40	71		.033
9-	45	46	90	55	40	61	.026	.069
16-	73	75	78	94	56	56	.014	.085

Table 1 Continued

Summary of Test Results and Computed Data

Week	Drum No.	No. Piles Length Spacing	P_s (#)	P_g (#)	P_I (#)	P_m (#)	$\bar{P}_g$ (#)	$\bar{P}_{cg}$ (#)
11	1	2-24-1.75	12.75	21.15	16.61	16.40	10.58	19.51
"	"	4-	12.75	40.33	16.61	16.40	10.08	14.64
"	2	9-	12.75	90.07	14.21	13.89	10.01	9.60
12	1	2-36-1.75	15.75	28.25	21.93	21.14	14.13	24.73
"	"	4-	15.75	56.33	21.93	21.14	14.08	18.76
"	2	9-	15.75	115.05	21.93	21.14	12.78	14.25
10	3	16-12-2.50	7.75	104.92	7.99	8.22	6.56	8.00
11	3	16-24-2.50	12.75	174.92	17.22	17.04	10.93	14.28
12	3	16-36-2.50	20.75	279.92	26.34	25.70	17.49	20.24

Table 1a Continued

Summary of Test Results and Computed Data

No. Piles Length Spacing	P_s/P_I (%)	P_s/P_m (%)	$\bar{P}_g/P_s$ (%)	$\bar{P}_g/\bar{P}_{cg}$ (%)	$\bar{P}_g/P_I$ (%)	Converse Labarre Eff. (%)	Single Pile Settlement (in.)	Single Pile Settlement (in.)
16-12-2.50	97	94	85	82	82	64	.022	.044
16-24-2.50	74	75	86	77	63	64	.017	.055
16-36-2.50	79	81	84	86	66	64	.013	.070
2-12-3.00	85	82	93	50	79	90	.012	.015
4-			83	47	70	80		.032
9-	103	100	74	56	77	73	.022	.045
16-	97	93	83	69	81	69	.016	.045
2-24-3.00	70	71	95	43	66	90	.011	.019
4-			86	45	60	80		.020
9-	79	80	73	50	58	73	.014	.055
16-	82	82	61	49	50	69	.014	.055
2-36-3.00	57	59	92	35	52	90	.015	.014
4-			88	39	51	80		.025
9-	63	65	82	48	52	73	.018	.044
16-	68	70	70	51	48	69	.015	.065

Table 2a

Bearing Capacity Comparison with the Meyerhof Analysis

No. Piles Length Spacing	P_g (#)	P_{mg} (#)	P_g/P_{mg} (%)
4-12-1.50	24.00	16.93	142
9-12-1.50	52.10	46.41	112
16-12-1.50	81.70	91.18	90
4-24-1.50	39.00	30.16	129
9-24-1.50	76.50	78.97	97
16-24-1.50	126.70	145.71	87
4-36-1.50	51.00	46.94	109
9-36-1.50	96.50	98.11	98
16-36-1.50	171.70	158.77	108
4-12-1.75	23.33	24.73	94
9-12-1.75	50.07	46.82	107
4-24-1.75	40.33	38.92	104
9-24-1.75	90.01	75.20	120
4-36-1.75	56.33	49.20	114
9-36-1.75	115.05	108.11	106

Table 2b

Depth vs Shear Strength (psi)

Drum No.	Depth 2"	8"	14"
I	0.704	0.704	0.704
II	0.704	0.735	0.796
III	0.704	0.742	0.789

Table 3a

Results of the Vane Shear Test

	Shear Strength (psi)											
Week	1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th	11th	12th
Drum I	0.856	0.856	0.826	0.826	0.796	0.765	0.735	0.735	0.795	0.887	0.796	0.700
Drum II	0.856	0.856	0.887	0,796	0.826	0.796	0.825	0.856	0.765	0.673	0.674	0.704
Drum III	0.856	0.796	0.735	0.918	0.918	0.888	0.916	0.949	0.795	0.735	0.827	0.856

Table 3b

Size of the Pile Groups

	Outside Dimensions of the Pile Groups (inches)			
Spacing (diameters)	1.50	1.75	2.00	3.00
Pile Groups				
2	1.25 x 0.50	1.38 x 0.50	1.50 x 0.50	2.00 x 0.50
4	1.25 x 1.25	1.38 x 1.38	1.50 x 1.50	2.00 x 2.00
9	2.00 x 2.00	2.25 x 2.25	2.50 x 2.50	3.50 x 3.50
16	2.74 x 2.75	4.25 x 4.25 *	3.50 x 3.50	5.00 x 5.00

* = Spacing of 2.50 diameters

Table 4

Typical Data Sheet

Single Pile		9 Pile Group		Vane Shear	
Load (#)	Settlement (in.)	Load (#)	Settlement (in.)	Load (gm)	Rotation (degree)
0.33 *	.000	2.27 *	.000	0	0
2.33	.002	12.27	.002	500	1
4.33	.004	22.27	.004	700	1 1/2
5.33	.005	32.27	.010	900	2
6.33	.006	42.27	.014	1100	3
7.33	.007	52.27	.019	1200	3 1/2
8.33	.008	57.27	.022	1300	4
9.33	.010	62.27	.027	1350	Failed
10.33	.011	67.27	.029		
11.33	.012	72.27	.034		
12.33	.014	77.27	.039		
13.33	Failed	82.27	.055		
		87.27	Failed		

* Initial load is equal to the weight of the pile cap plus the weight of the portions of the piles that are not embedded.

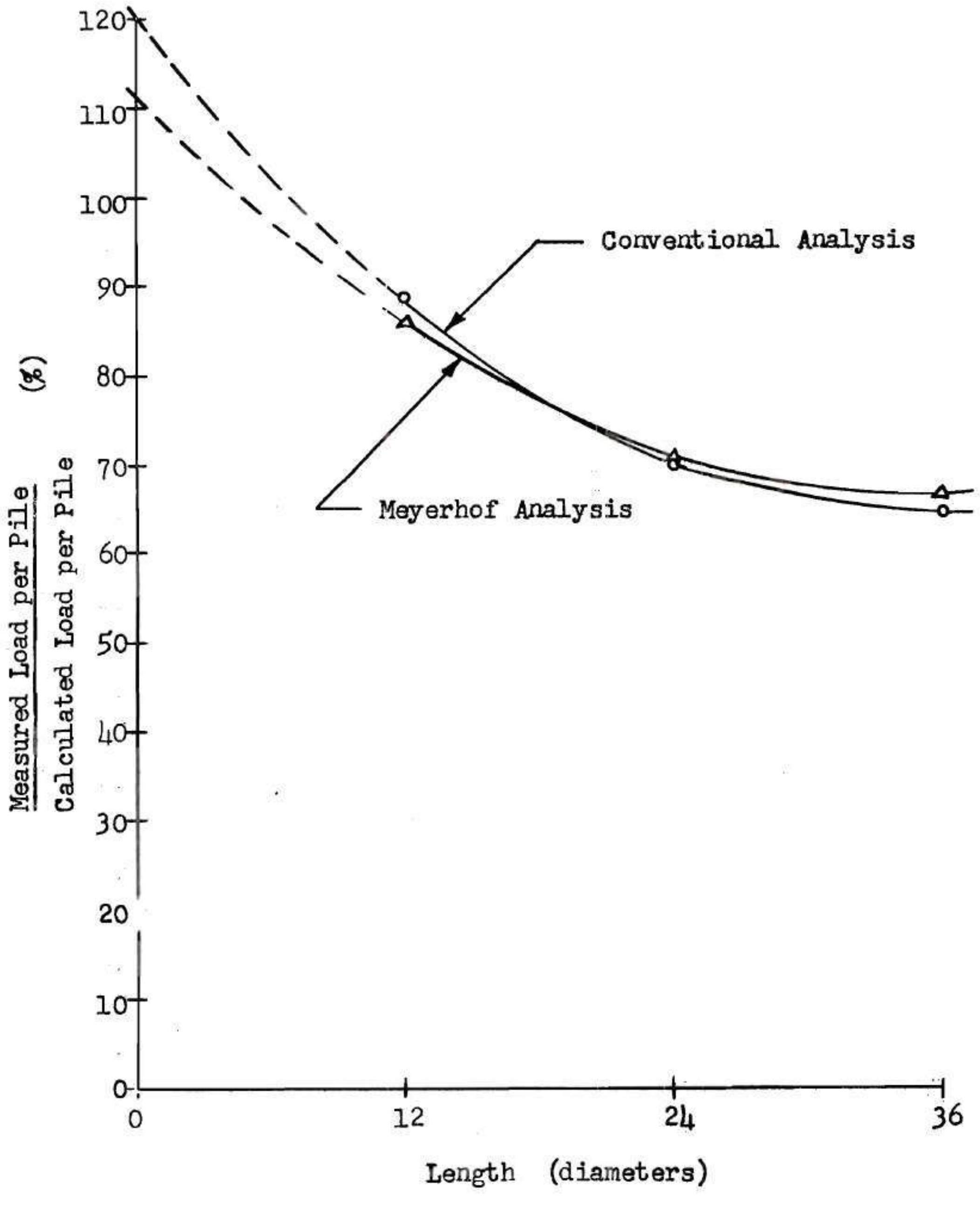
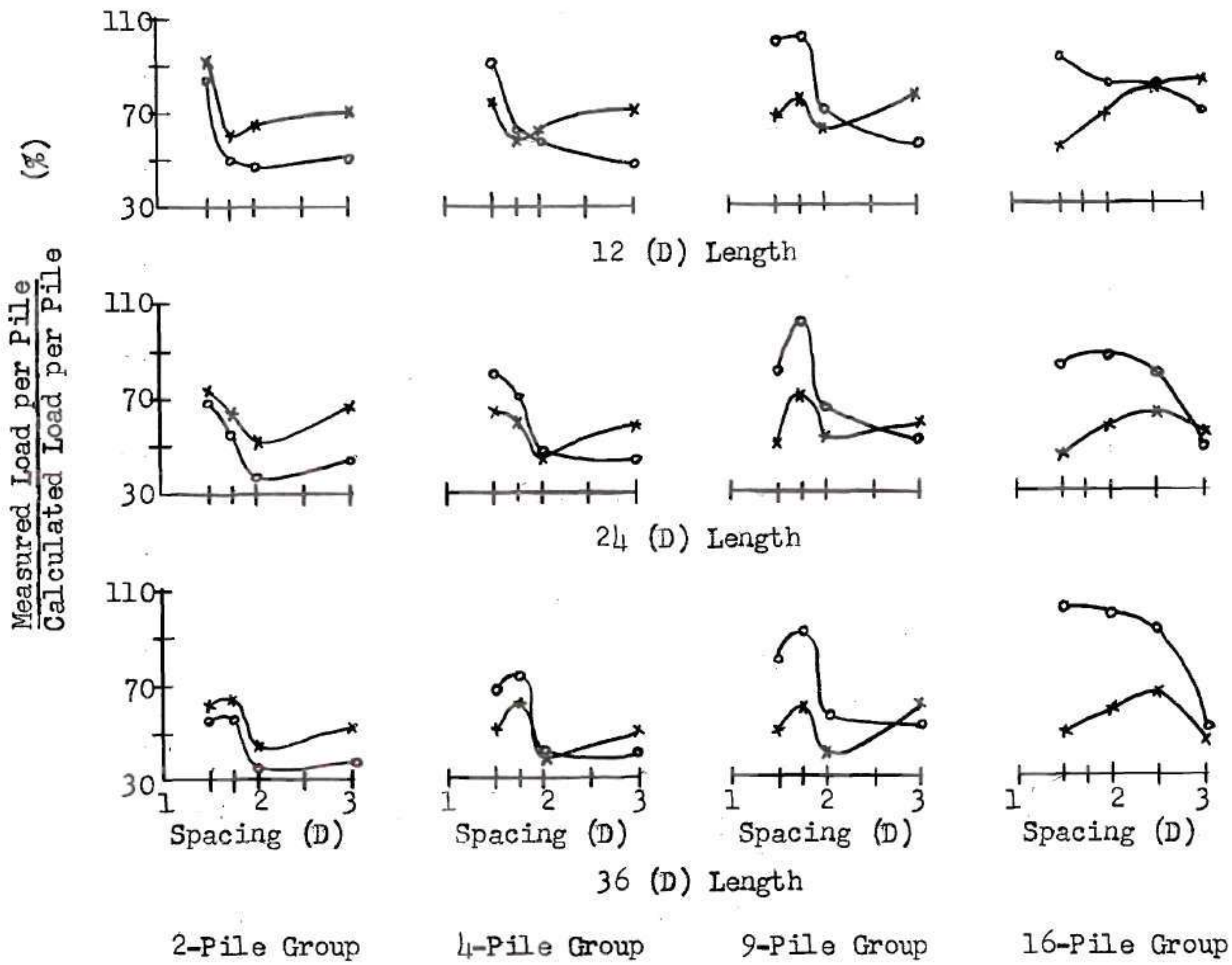


Fig. 1. The Average Ratio of Measured Load per Pile to the Calculated Load per Pile Compared to the Pile Length



x = Individual Action Efficiency

o = Unit Action Efficiency

Fig. 2. Theoretical Efficiencies vs Pile Spacing

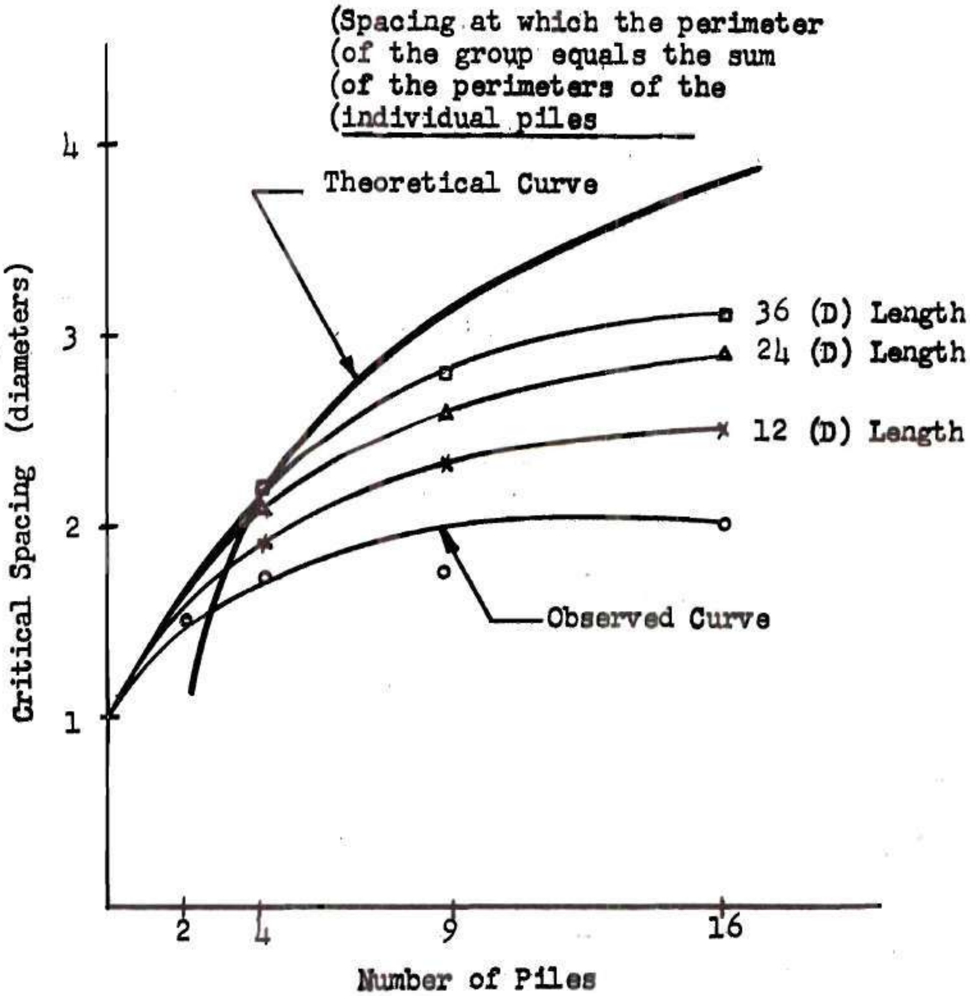
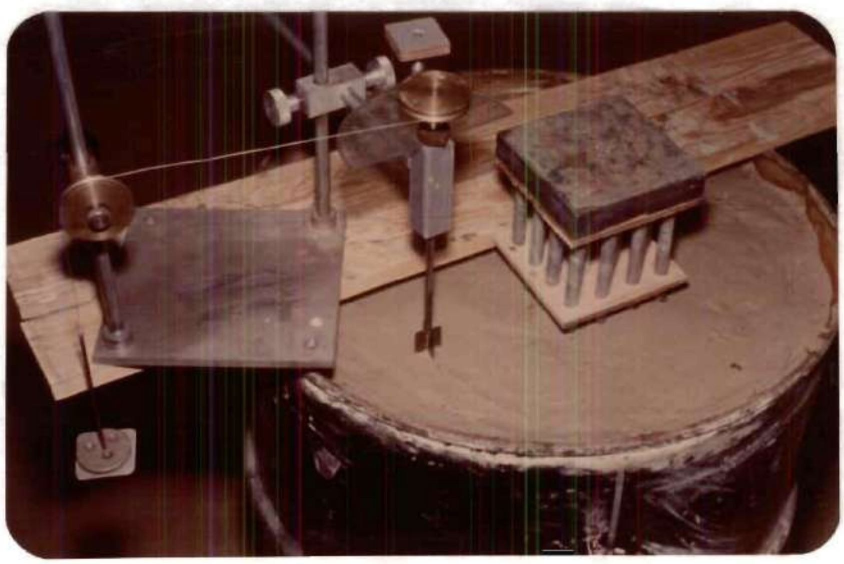


Fig. 3. Critical Spacing vs Number of Piles

-a-

An Example of a Unit Action Failure.



-b-

Vane Shear Device

Fig. 4.

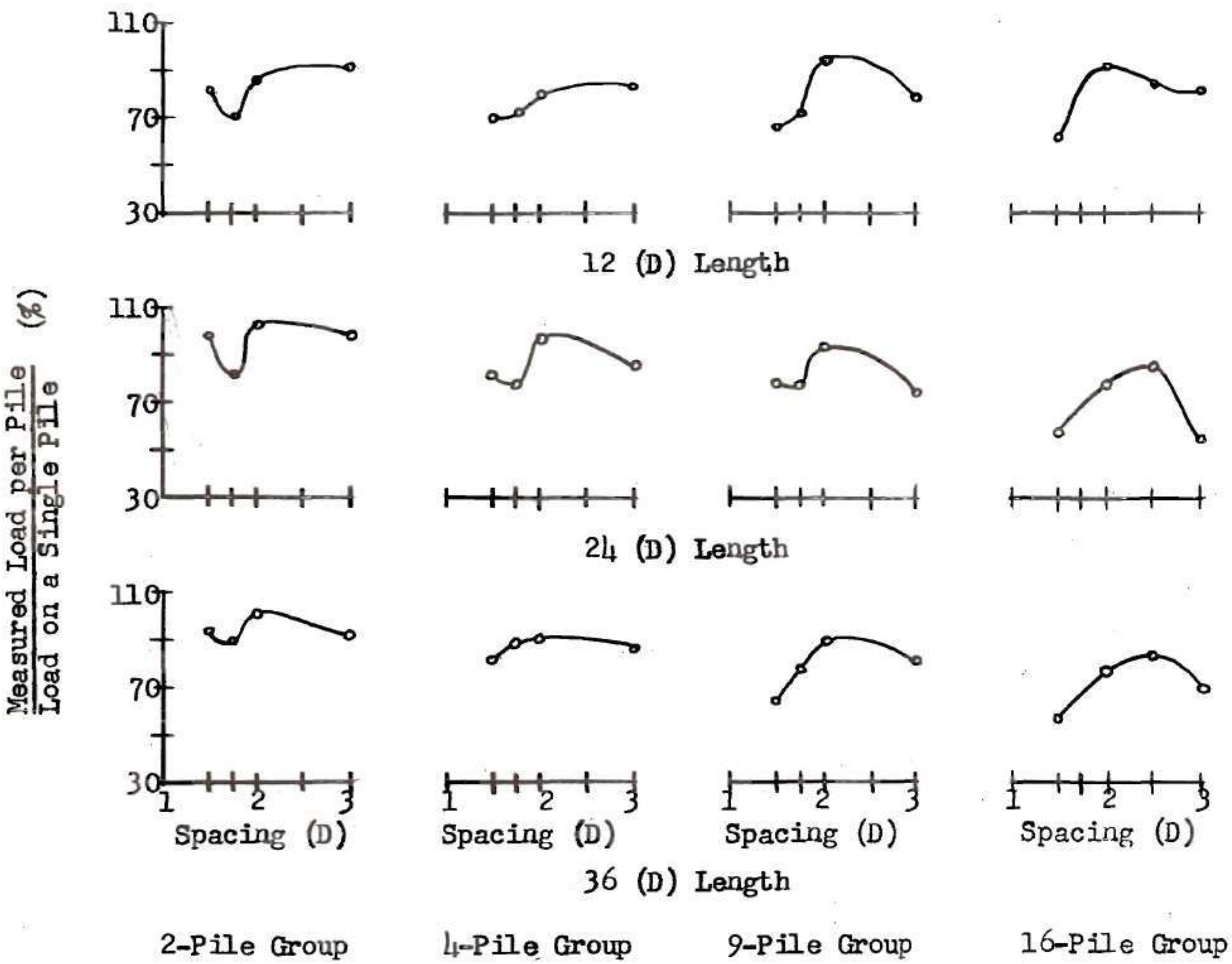


Fig. 5. Single Pile Efficiency vs Spacing

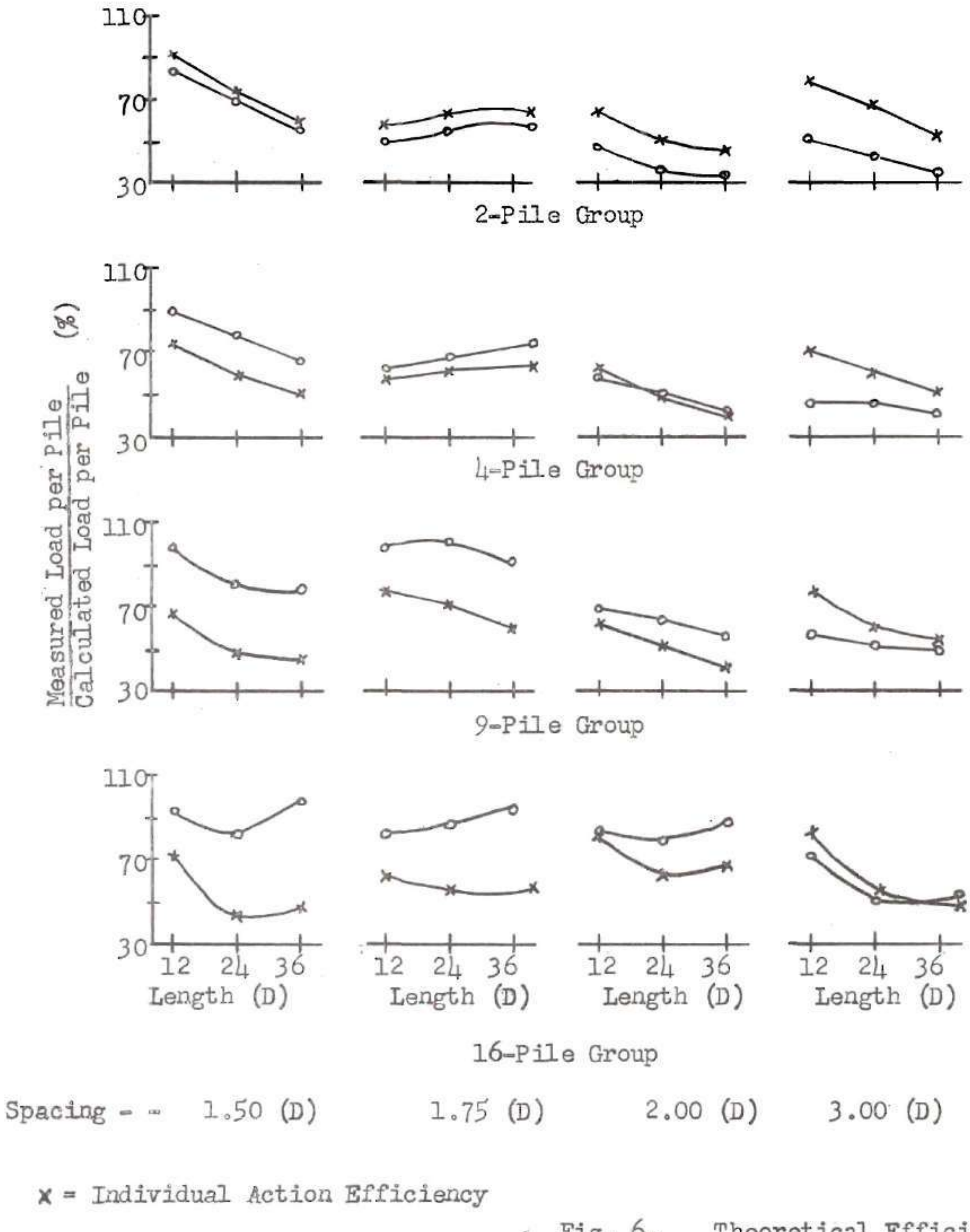


Fig. 6. Theoretical Efficiencies vs Pile Length

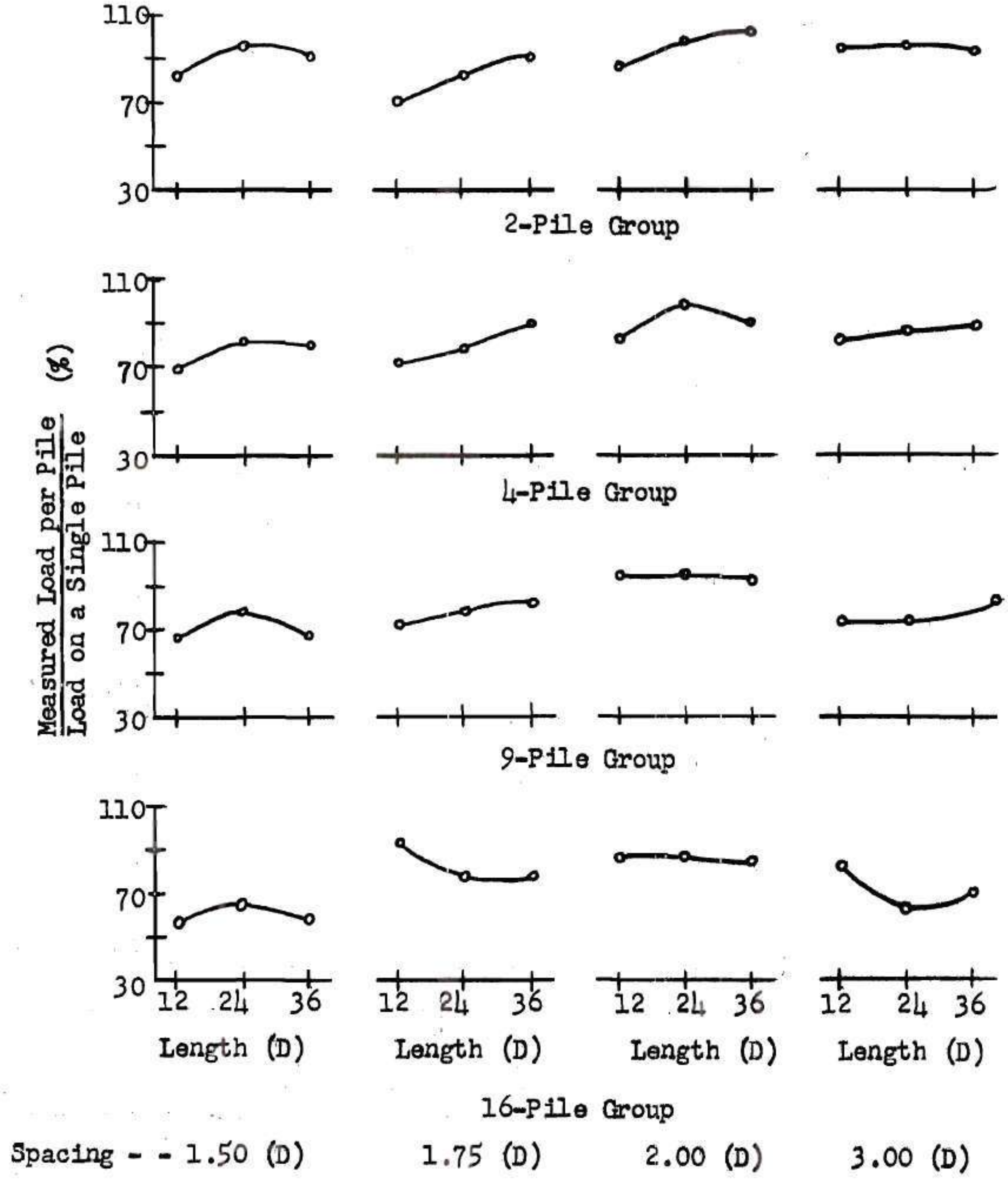
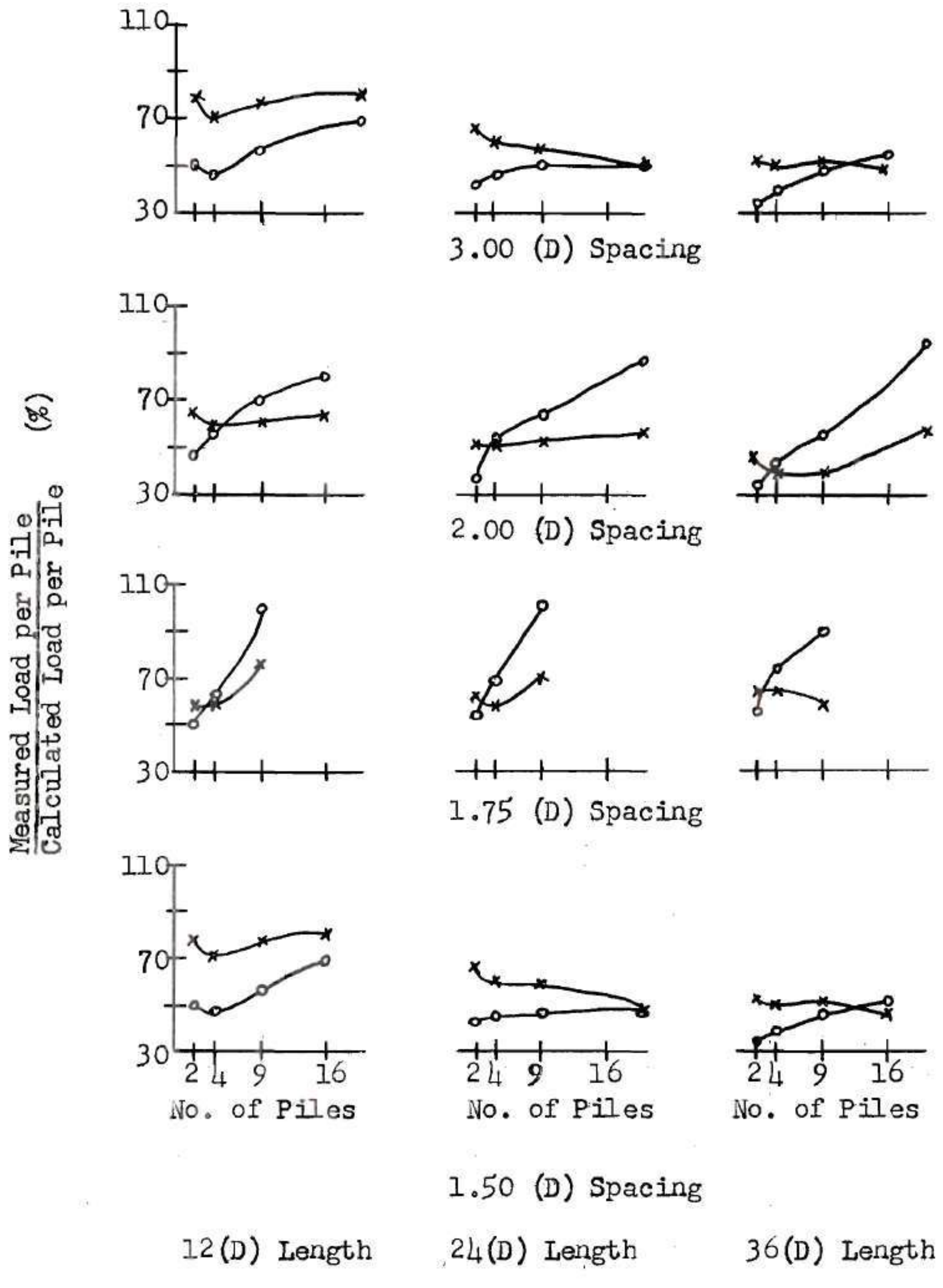


Fig. 7. Single Pile Efficiency vs Pile Length



x = Individual Action Efficiency

o = Unit Action Efficiency

Fig. 8. Theoretical Efficiencies vs Number of Piles

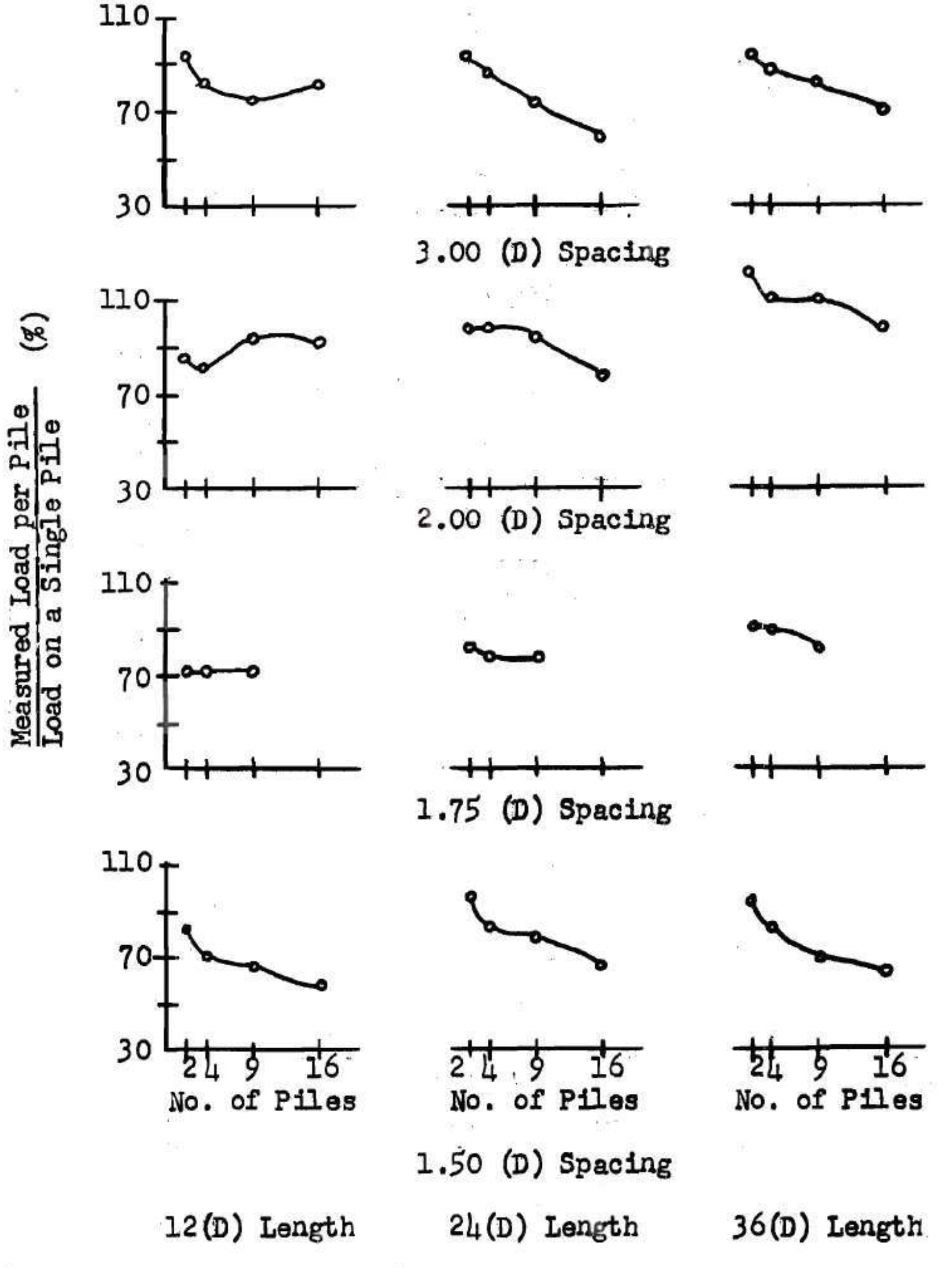
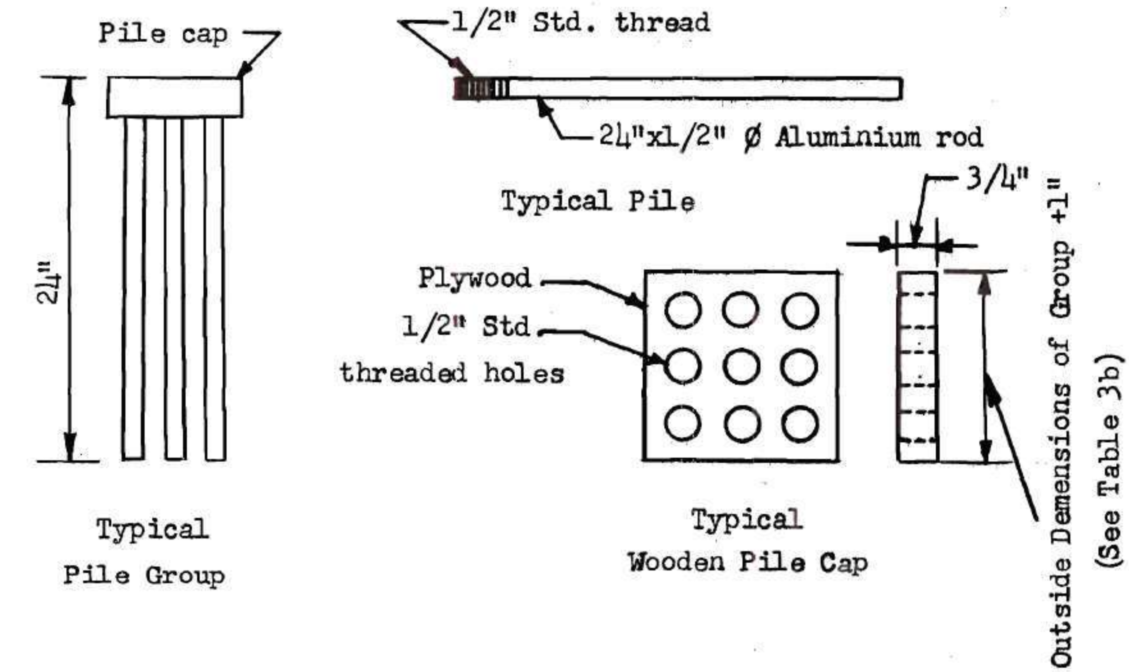


Fig. 9. Single Pile Efficiency vs Number of Piles



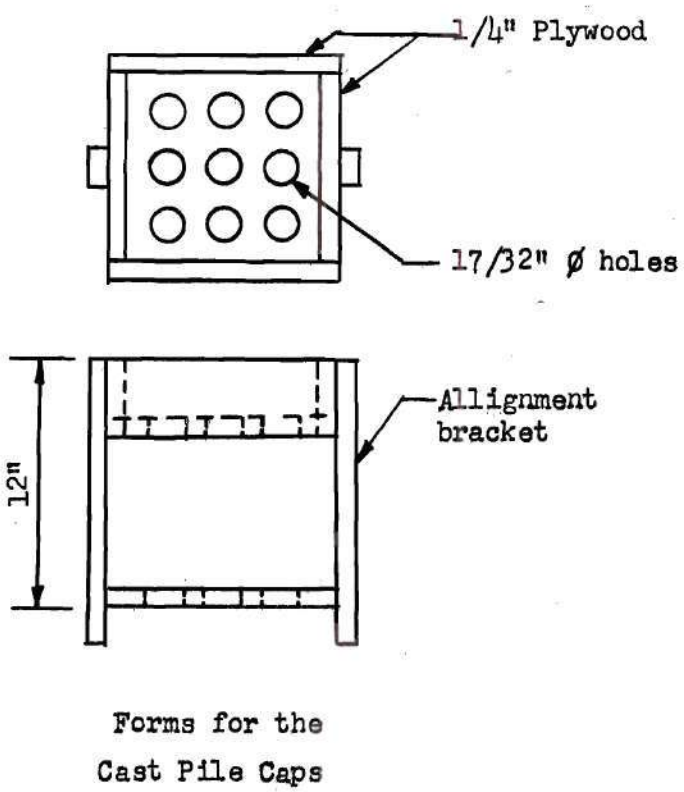
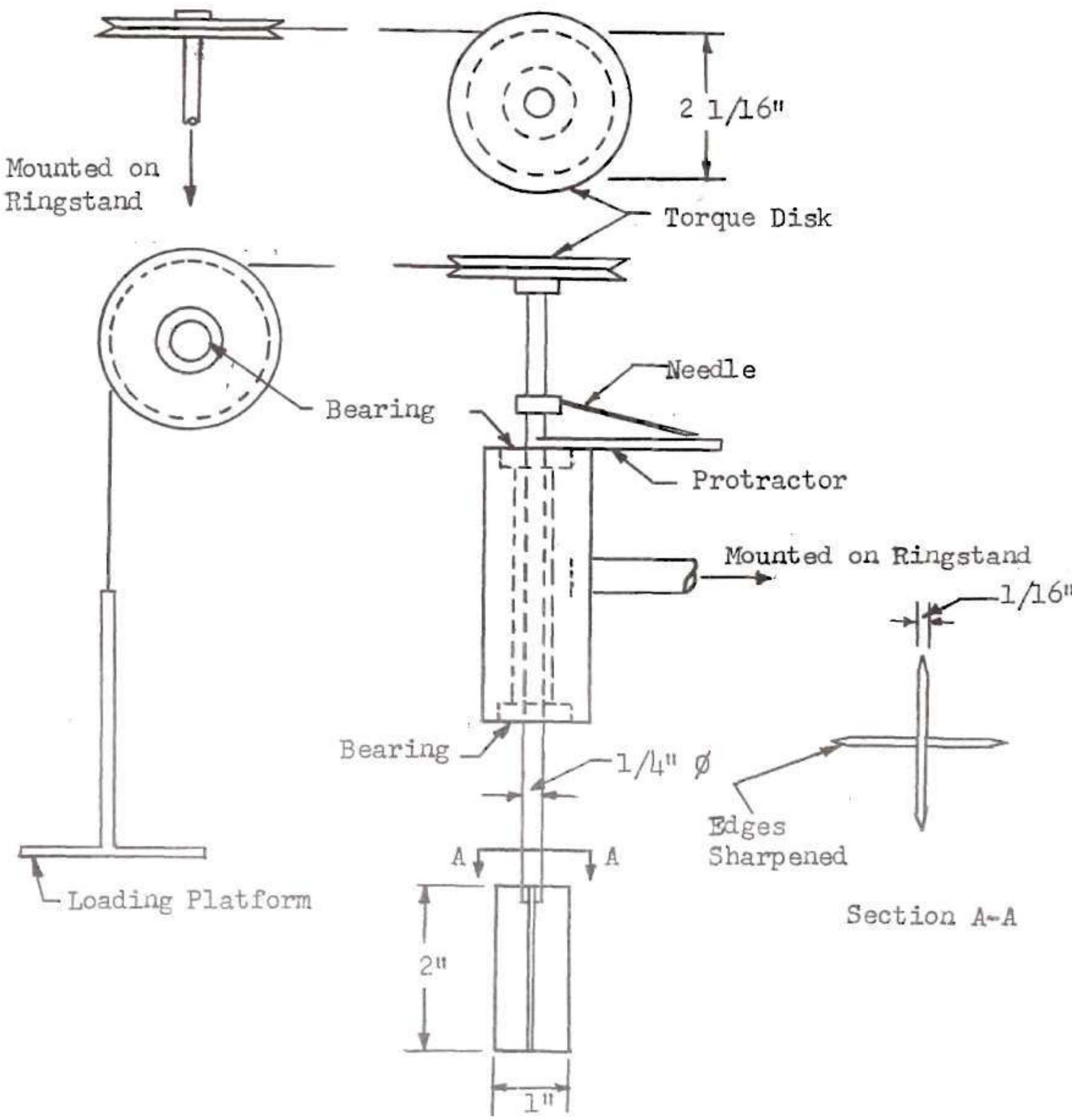


Fig. 10.

Detains of a Typical Pile Group



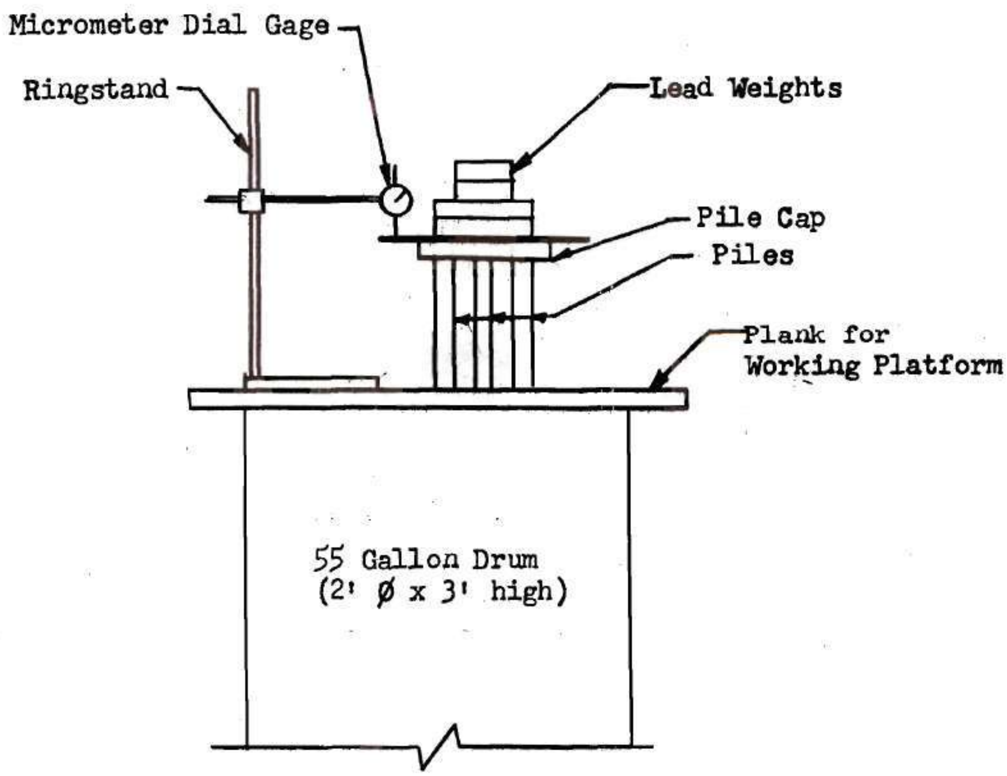
Shear Strength = $\dfrac{Pa}{\pi a^2 (L + a/3)}$

where P = Critical load
d = Ø of torque disk
a = Diameter of vane
L = Heigth of vane

Fig. 11

Vane Shear

Apparatus



Approximate Scale: 1" = 1'

Fig. 12

Setup for Loading Piles

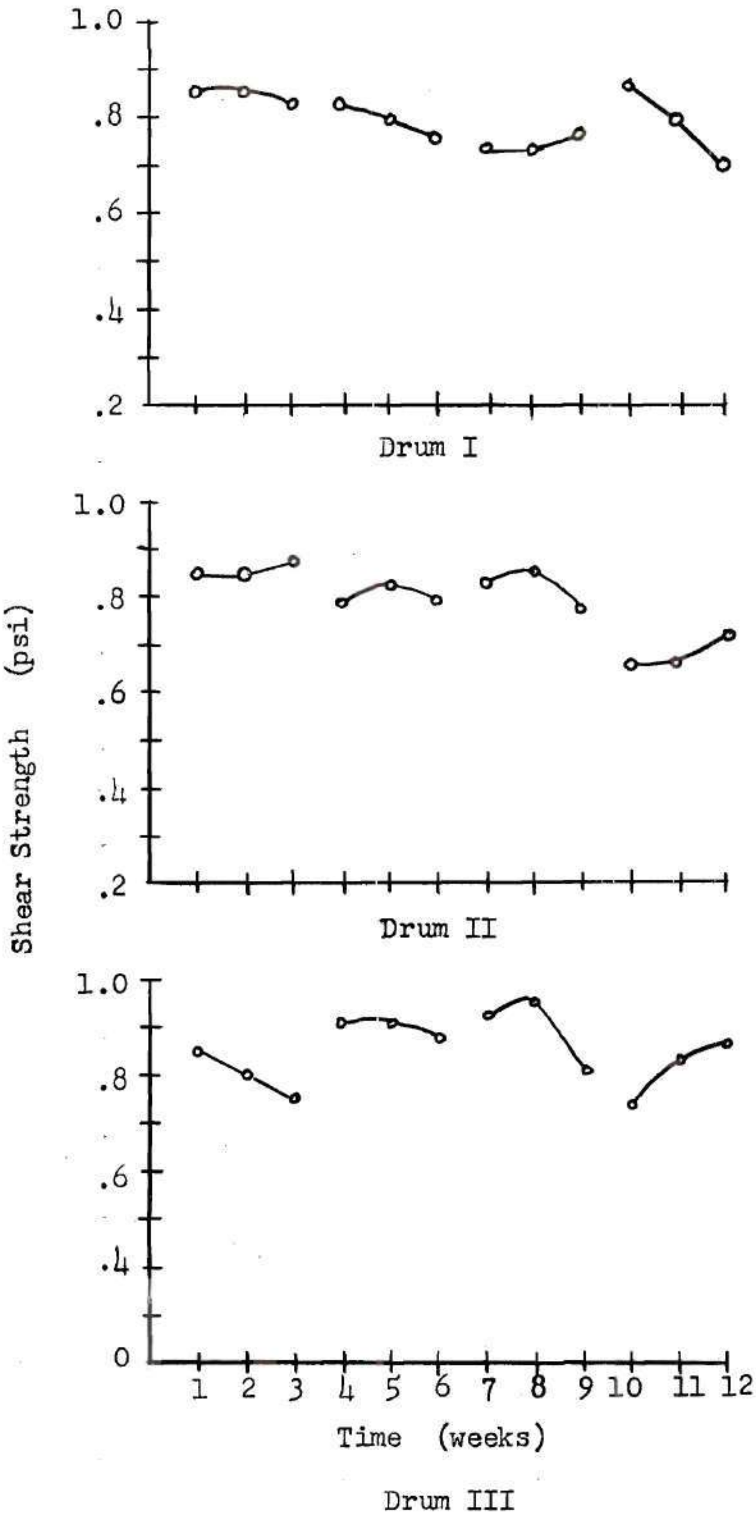
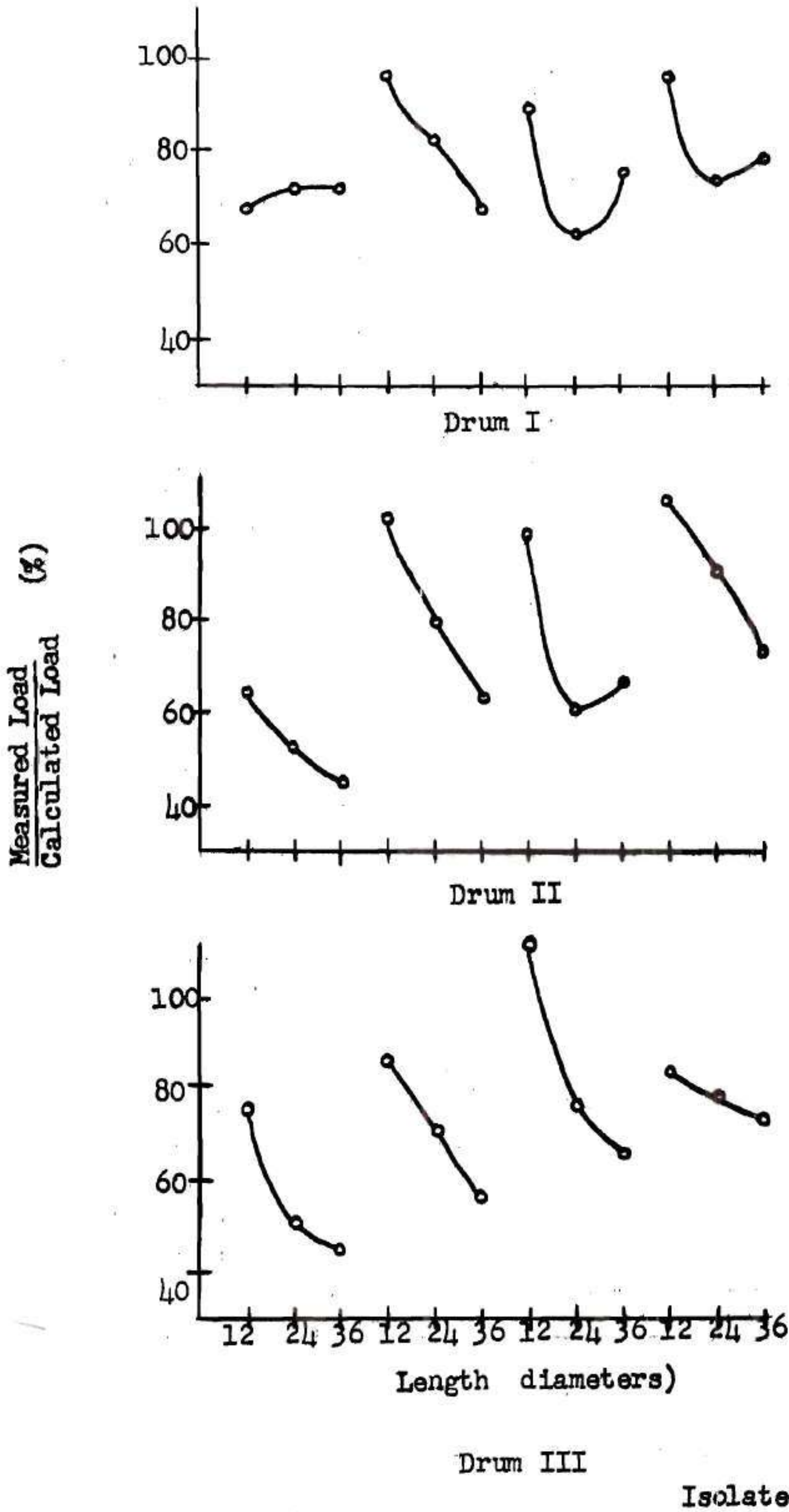


Fig. 13. Results of the Vane



Isolated Pile Efficiencies vs Length

Fig. 14.

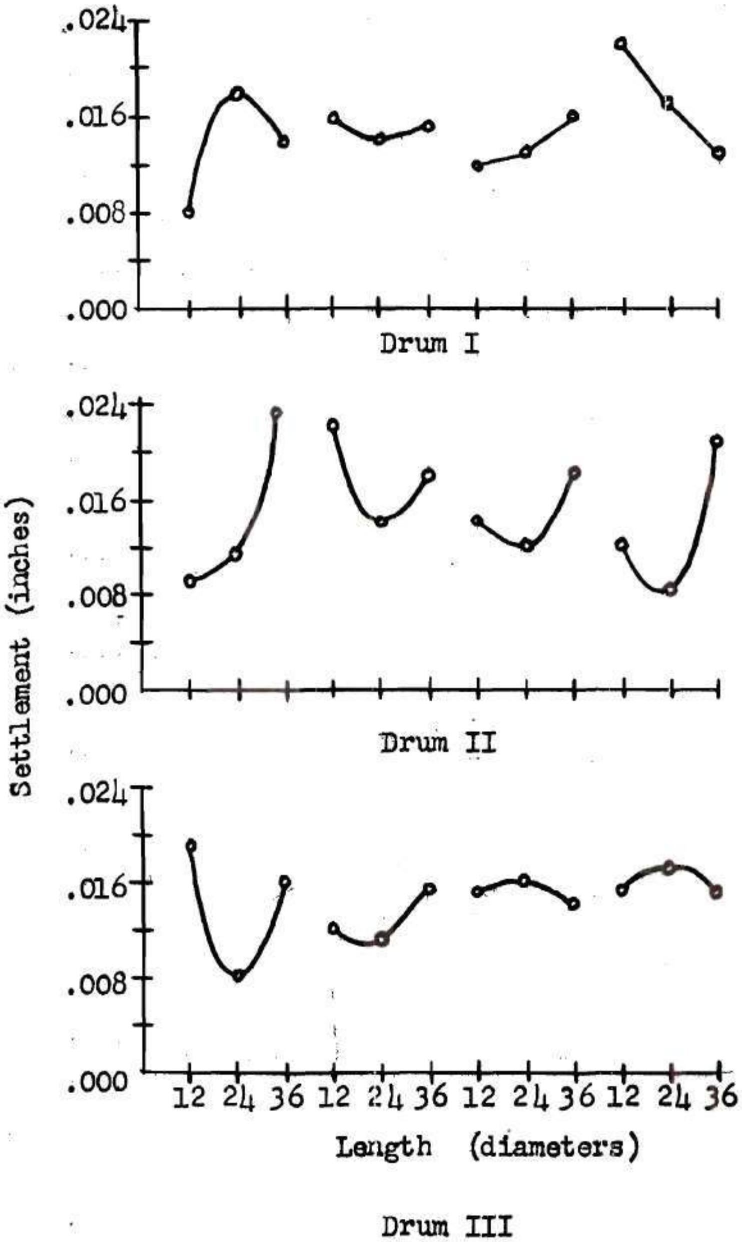


Fig. 15. Isolated Pile Settlements

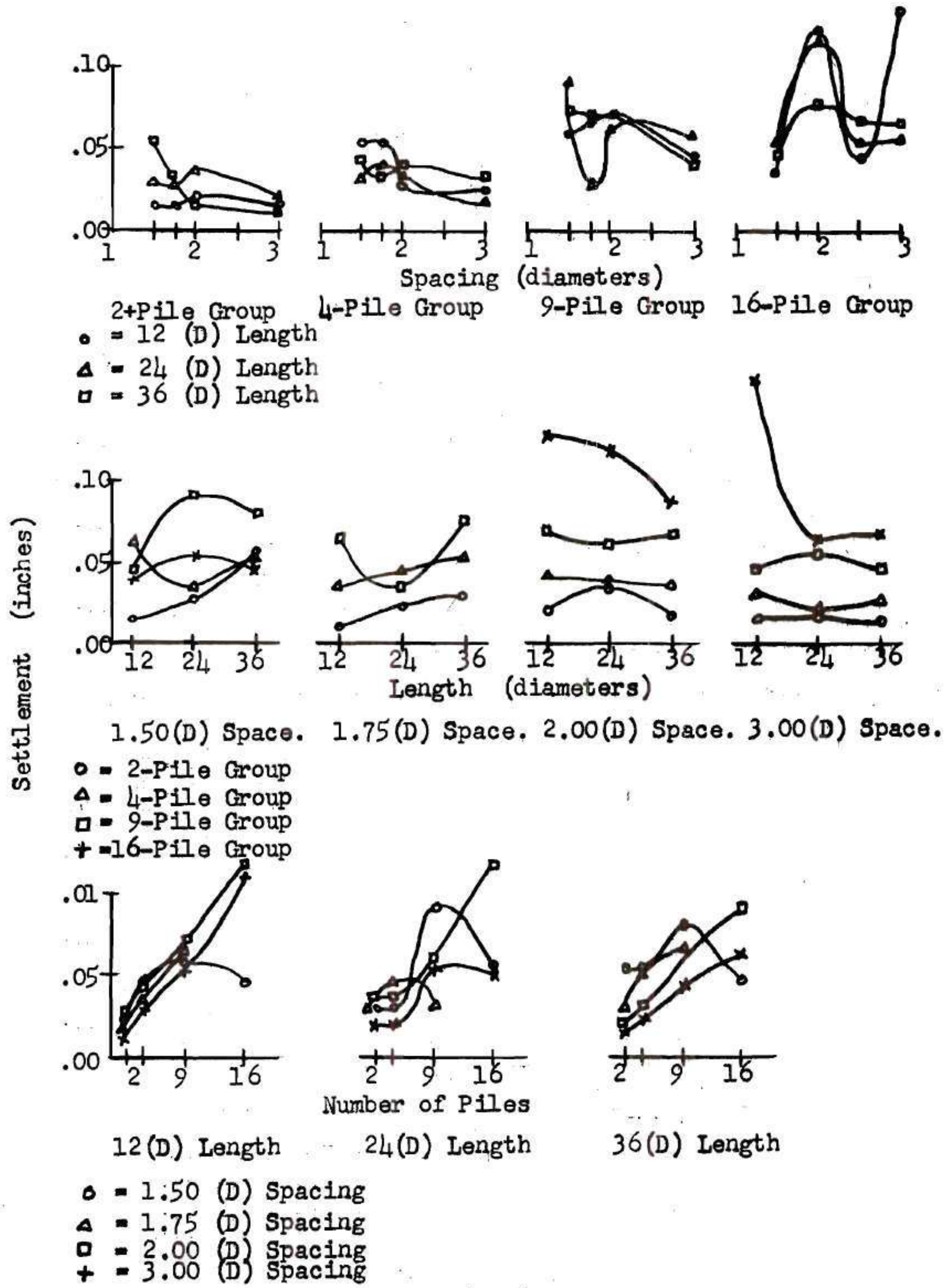


Fig. 16. Pile Group

Settlements

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